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Evaluation of Seismic performance of RC setback frames

Alireza Habibi[1a], Meisam Vahed[2b] and Keyvan Asadi[*2]

[1]*Department of Civil Engineering, Shahed University, The beginning of the Tehran-Qom Highway, 3319118651, Tehran Province, Tehran, Iran*
[2]*Department of Civil Engineering, University of Kurdistan, Kurdistan Province, Sanandaj, Pasdaran Street, 6617715175, Iran*

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Abstract. When the irregularities occurred in buildings, affect their seismic performance. This paper has focused on one of the types of irregularities at the height that named setback in elevation. For this purpose, several multistorey Reinforced Concrete Moment Resisting Frames (RCMRFs) with different types of setbacks were designed according to new edition of Iranian seismic code. The nonlinear time history analysis was performed to predict the seismic performance of frames subjected to seven input ground motions. The assessment of the seismic performance was done considering both global and local criteria. Results showed that the current edition of Iranian seismic code needs to be modified in order to improve the seismic behaviour of reinforced concrete moment resisting setback buildings. It was also shown that the maximum damages happen at the elements located in the vicinity of the setbacks. Therefore, it is necessary to strengthen these elements by appropriate modification of Iranian seismic code.

Keywords: RC buildings; Irregularity in elevation; Setback frame; Seismic performance; Standard 2800

1. Introduction

Nowadays, Vertical irregularities in buildings have become common due to architectural views and functional requirements. One type of vertical geometrical irregularity in building structures is the presence of setbacks that the plan dimensions suddenly change over the height of the building. Variations of the mass and stiffness of setback buildings change the dynamic characteristics as compared to regular ones. Experiences from the past earthquakes have shown that setback buildings exhibit inadequate behaviour in spite of being designed according to the seismic codes requirements at the time. Therefore, some studies discussed the adequacy of simplified seismic code design procedures when they are applied to this type of buildings. The first one performed by Penzien and Chopra (1965) that presented an approximate method for evaluating the response of these buildings and Penzien (1969) later extended his method. After that Pekau and Green (1974) and Humar and Wright (1977) investigated seismic response of setback structures and observed that inter storey drift demands were increased near the location of the setback level. Aranda (1984) concluded that the ductility demands in setback structures are higher than those of regular structures. Shahrooz and Moehle (1990) observed concentration of inelastic behaviour in members near setback. Wood (1992) observed no difference in the seismic response of setback and regular structures. Wong and Tso (1994) studied the elastic response of setback structures and found that the first mode is capable of representing the displacement response. Duan and Chandler (1995) showed that the torsional response is recognized as one of the important causes of damage to one-side setback buildings during strong earthquakes. The main conclusion of performed research by Mazzolani and Piluso (1996) is that the presence of setbacks does not lead to significant worsening of the seismic responses. Chen *et al.* (2000) observed the damage concentration to be greater at the tower portion of the setback. Romão *et al.* (2004) found that setback buildings exhibit an adequate seismic performance when compared to the regular ones. Tena-Colunga (2004) by considering slender setback frames designed based on Mexican seismic code concluded that seismic codes should penalize seismic design of slender setback buildings with single bay frames in one direction. Khoury *et al.* (2005) observed amplification in response and excessive damage concentration to the setback frames at the upper stories in tower portion. Lignos and Gantes (2005) showed that the modal pushover analysis (MPA) cannot predict the collapse of frames with stiffness irregularities. DeStefano *et al.* (2005) by considering a set of plane vertically irregular RC frames designed according to EC8 revealed that the P-Δ effect has a significant influence on seismic performance of these types of structures. Athanassiadou (2008) assessed the seismic performance of RC setback frames designed according to EC8 and concluded this code provides the satisfactory seismic performance for setback frames. Karavasilis *et al.* (2008) based on the results of parametric study conducted on a large number of steel frames, observed the maximum deformation demands are concentrated in the tower for tower-like structures and in the neighborhood of the setbacks for other geometrical configurations. Kappos and

*Corresponding author, M.Sc. Student
E-mail: asadikeyvan@yahoo.com
[a]Ph.D.
E-mail: ar.habibi@shahed.ac.ir
[b]M.Sc. student
E-mail: vahed.ac@yahoo.com

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Stefanidou (2010) presented a new method for evaluating the seismic responses of reinforced concrete setback frames and revealed the advantages of the proposed design method, especially more economic detailing of transverse reinforcement in the members. Sarkar *et al.* (2010) proposed a new method of quantifying irregularity in Stepped building frames, accounting for dynamic characteristics (mass and stiffness). In addition, they developed an empirical formula to calculate the fundamental time period of building frames with vertical setbacks. Georgoussis (2011) investigated the dynamic response of tall setback buildings and proposed a method for assessing vibration frequencies and modal base shears and torques. Result showed that the vertical mass irregularity increases substantially the contribution of the higher modes of vibration to the total response of such structures. Varadharajan *et al.* (2012) summarized the researches done in the past regarding different types of structural irregularities. They expressed that the strength irregularity has the maximum impact and mass irregularity has the minimum impact on seismic responses of vertical irregular structures. Shakib and Pirizadeh (2014) investigated the seismic performance of one-side setback structures, designed based on Iranian seismic code (3rd edition) with a probabilistic approach. They demonstrated the poor performance of these torsionally coupled structures and necessity of the revision of the seismic code provisions for geometric vertical irregularities. Rahami *et al.* (2013) proposed a method for the analysis of irregular structures in the form a regular structure and showed capability of their proposed method for several irregular structures. Varadharajan *et al.* (2013) aimed to determine the effect of setback on inelastic deformation demands by considering an extensive parametric study on plane RC setbacks frames. Results indicated strong influence of the parameters like beam-column strength ratio, number of stories, geometrical irregularity and the performance level under consideration on inelastic seismic demands. Wu *et al.* (2013) by considering a rigid-connected twin-tower skyscraper with asymmetrical distribution of stiffness and masses in two towers concluded that the Chinese code can provide the goal of no damage under frequent earthquakes and no collapse under rare earthquakes. Zahid *et al.* (2013) used the nonlinear static analysis to investigate the overstrength factor of reinforced concrete regular and irregular frames designed according to EC2 and EC8 and concluded that the geometry and ductility of the frames affect the overstrength factor. Habibi and Asadi (2014) by performing nonlinear dynamic time-history analysis on several regular and irregular RC frames designed according to 3rd Edition of Iranian seismic code concluded that when setback occurs in elevation, the requirements of the life safety level are not satisfied and the revision of the Iranian seismic code for irregular frames with setback is necessary. Landi *et al.* (2014) investigated the effectiveness of several conventional, multi-modal and adaptive pushover procedures for estimating seismic demand of RC regular and irregular frames. Their results showed that the advanced procedures, in particular the multi-modal pushover, provided an improvement of the results, more

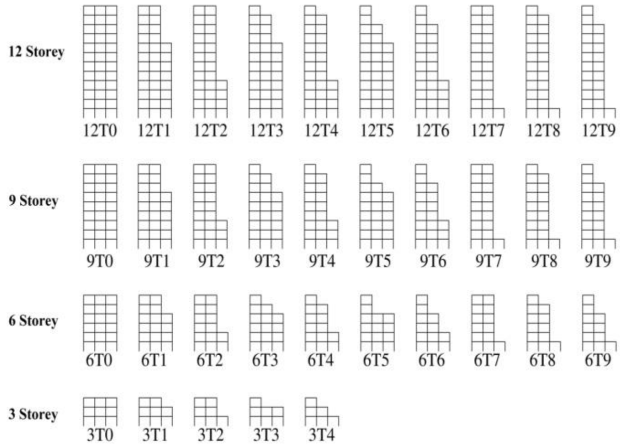


Fig. 1 Studied structures. (Habibi and Asadi 2014)

evident for the irregular frames. Varadharajan *et al.* (2014) proposed some equations for quantifying the setback irregularity for the fundamental period of vibration and for estimating the maximum inter storey drift ratio and maximum displacement ductility. The proposed equations were validated for 2D and 3D building models with setback irregularity. Bigdeli *et al.* (2014) developed a new nonlinear model for active control of three-dimensional (3D) irregular building structures. They verified the proposed control system and training algorithm of the structural system, by simulating the responses of the structure under the El-Centro 1940 earthquake excitation and showed that the proposed method is effective in structural control. Zhou *et al.* (2015) evaluated the seismic control effect of vertical irregularity factors for RC structures using the Monte Carlo simulation method. The results indicated that the exceeding probability increases as the vertical irregularity factor decreases. A minimum strength and stiffness irregularity factor of 0.7 were proposed as the seismic control limit. Mazza *et al.* (2015) proposed a displacement based design procedure for proportioning hysteretic damped braces (HYDBs) in order to attain, for a specific level of seismic intensity, a designated performance level of a reinforced concrete irregular in elevation framed building which has to be retrofitted. Gürsoy, *et al.* (2015) investigated the effects of weak storey irregularity on earthquake behaviour and rough construction costs of RC buildings designed based on the Turkish Earthquake Code. Varadharajan *et al.* (2015) studied a family of 108 frames with mass irregularity and proposed a parameter for quantify the mass irregularity. Also, they aimed to modify the expression of fundamental period proposed by IS 1893:2002 and evaluated the relation between mass irregularity coefficient and fundamental period. Habibi and Asadi (2016) by the aid of inelastic dynamic time-history analysis on several reinforced concrete moment resisting setback frames, proposed two relations to estimate the Park-Ang damage index for setback frames by applying irregularity indices. The effect of the vertical geometric irregularities on the fundamental periods of masonry infilled structures was investigated by Asteris *et al.* (2017). They proposed a reduction factor to quantify the reduction of the fundamental period due to the vertical setback irregularities.

Table 1 Ground motion records

No.	Earthquake	Station	PGA (g)	Normalized Factor	
				3,6&9-storey	12-storey
1	Manjil Iran, 6/20/1990	Abbar	0.514	0.893	1.03
2	Chuetsu-oki, 7/16/2007	Matsushiro Tokamachi	0.193	2.379	2.741
3	Kern County 1952/07/21	1095 Taft Lincoln School	0.177	2.587	2.980
4	Cape Mendocino 1992/04/25	Ferndale Fire	0.376	1.144	1.41
5	Northridge 1994/01/17	24157 LA - Baldwin Hills	0.238	1.925	2.22
6	Cape Mendocino 1992/04/25	89324 Rio Dell Overpass - FF	0.385	1.193	1.375
7	Northridge 1994/01/17	24538 Santa Monica City Hall	0.369	1.243	1.433

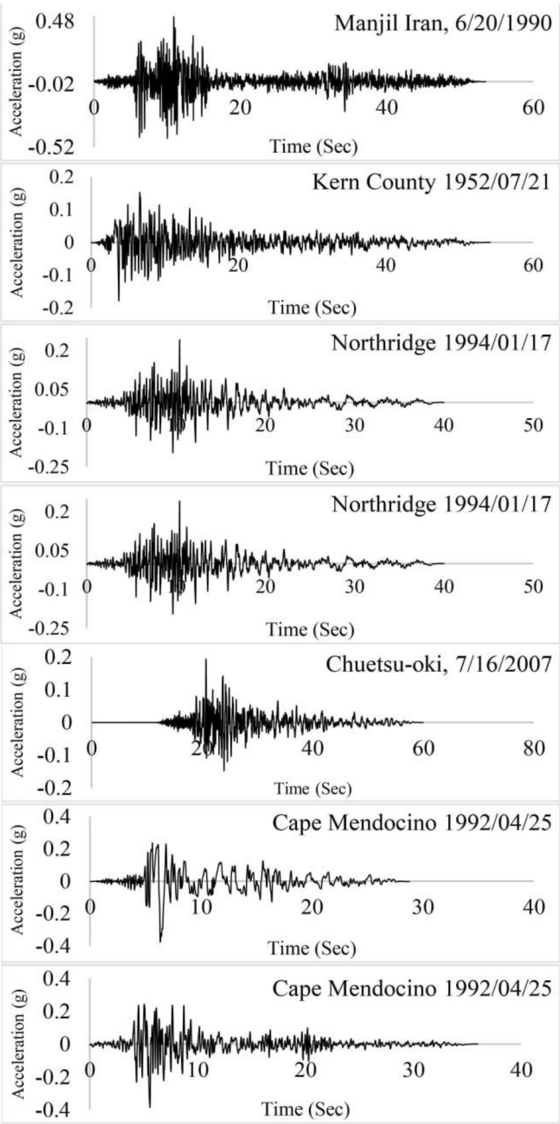


Fig. 2 Acceleration time histories of input records.

Aforementioned research revealed the significant affects of irregularity especially presence of setback on seismic performance of structures and accordingly the seismic code provisions must be evaluated for this type of irregular structures. Based on the studies of Shakib and Pirizadeh (2014) and Habibi and Asadi (2014), the 3rd edition of Iranian seismic code is not able to satisfy the requirements of the life safety level in the setback frames. Since the new edition of the Iranian seismic code has not been evaluated for setback structures, this paper by performing nonlinear dynamic analysis on several multistorey Reinforced Concrete Moment Resisting Frames (RCMRFs) with different types of setbacks, aims to evaluate seismic performance of these types of irregular structures designed according to the last edition of Iranian seismic code.

2. Studied structures and earthquake ground motions

In this study, thirty five plan reinforced concrete moment resisting frames as shown in the **Fi**g. 1 were designed according to new edition of Iranian national building's code and Iranian seismic code. The concrete cylinder strength of 30 Mpa, the steel yield strength of 400 Mpa, Soil type II and peak ground acceleration (PGA) of 0.35 g were assumed. All the frames have the storey height of 3.2 meters, and the bay length of 4 meters four of them are regular and other ones are irregular with different arrangements of setbacks. Irregular frames were designed with the aid of modal response spectrum analysis according to Standard 2800, whereas in the cases of the regular frames the (static) 'lateral force method of analysis' was used. The first four modes of vibration were considered in the multimodal analysis of all irregular frames, with total contributing masses more than 95% in all cases.

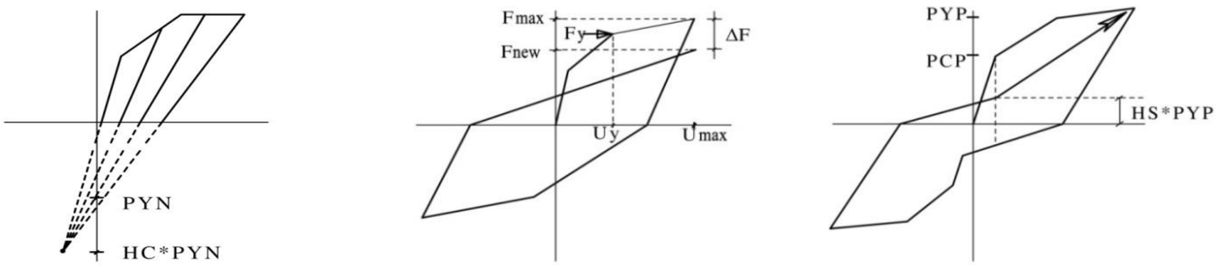
In order to perform nonlinear dynamic time-history analysis, seven strong ground motions were selected from Pacific Earthquake Engineering Research center (PEER) for soil type II and far from the causative fault as presented in Table 1. All records were normalized for design spectrum of standard No. 2800 for a PGA=0.35 g. The time-acceleration diagrams have been plotted in Fig. 2.

3. Structural modelling

In this paper, inelastic dynamic time-history analysis of all frames has been performed by the computer program IDARC Version 6.1. In the program IDARC, most structural elements, i.e. columns and beams, are modelled using the same basic macro formulation. Flexural, shear and axial deformations are considered in the general structural macro element, although axial deformations are neglected in the beam element. Flexural and shear components in the deformation are coupled in the spread plasticity formulation. When the member experiences inelastic deformations, cracks tend to spread form the joint interface resulting in a curvature distribution as shown in Fig. 3(a). Sections along the element will also exhibit different flexibility characteristics, depending on the degree of inelasticity observed. The flexibility distribution in the structural elements is assumed to follow the distribution shown in Fig. 3(b), where EI_A and EI_B are the current flexural stiffness of the sections at ends of the element; EI_0 is the stiffness at the center of the element; α_A and α_B are the yield penetration coefficients; and L is the length of the element. The moment curvature envelope describes the

(a) Curvature distribution along a RC element (b) flexibility assumption along a RC element

Fig. 3 Curvature distribution and flexibility assumption



(a) Model of stiffness degradation (b) Model of strength deterioration (c) Model of slip or pinching behaviour

Fig. 4 Control parameters for the three parameter hysteretic model

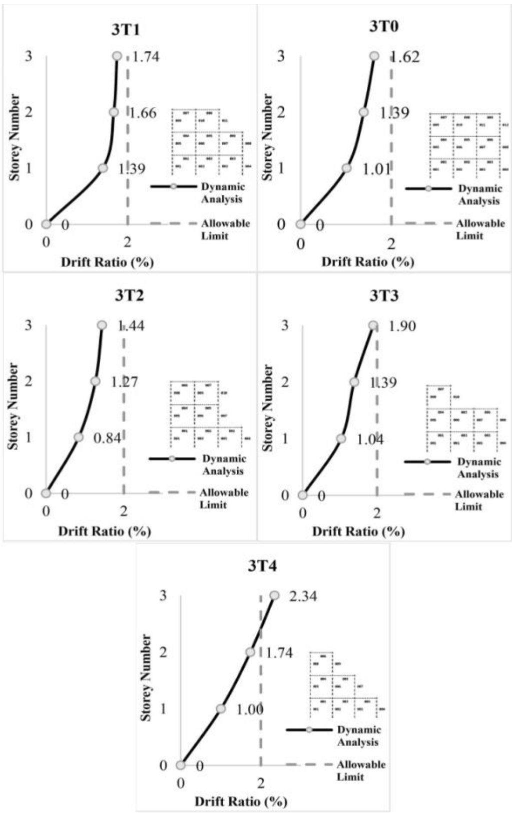


Fig. 5 Interstorey drifts ratios for 3 storey frames

changes in the force capacity with deformation during a nonlinear analysis. Therefore, the moment-curvature envelopes for columns and beams form an essential part of the analysis. The moment-curvature is internally determined by the program IDARC based on a fiber model analysis of the cross-section. Modelling the hysteretic behaviour of structural elements is one of the core aspects of a nonlinear structural analysis program. In this study, the elements of the structures are modelled using a three parameter Park

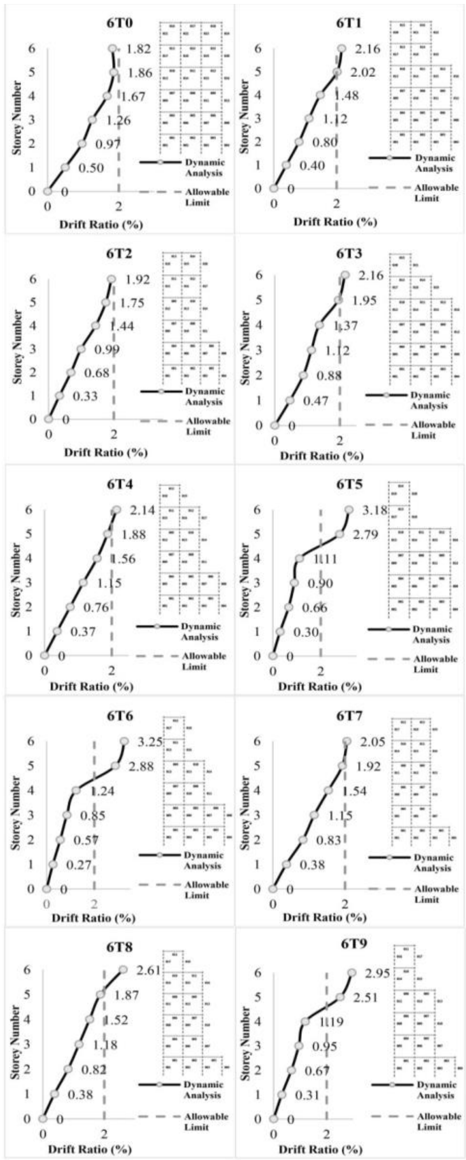


Fig. 6 Interstorey drifts ratios for 6 storey frames

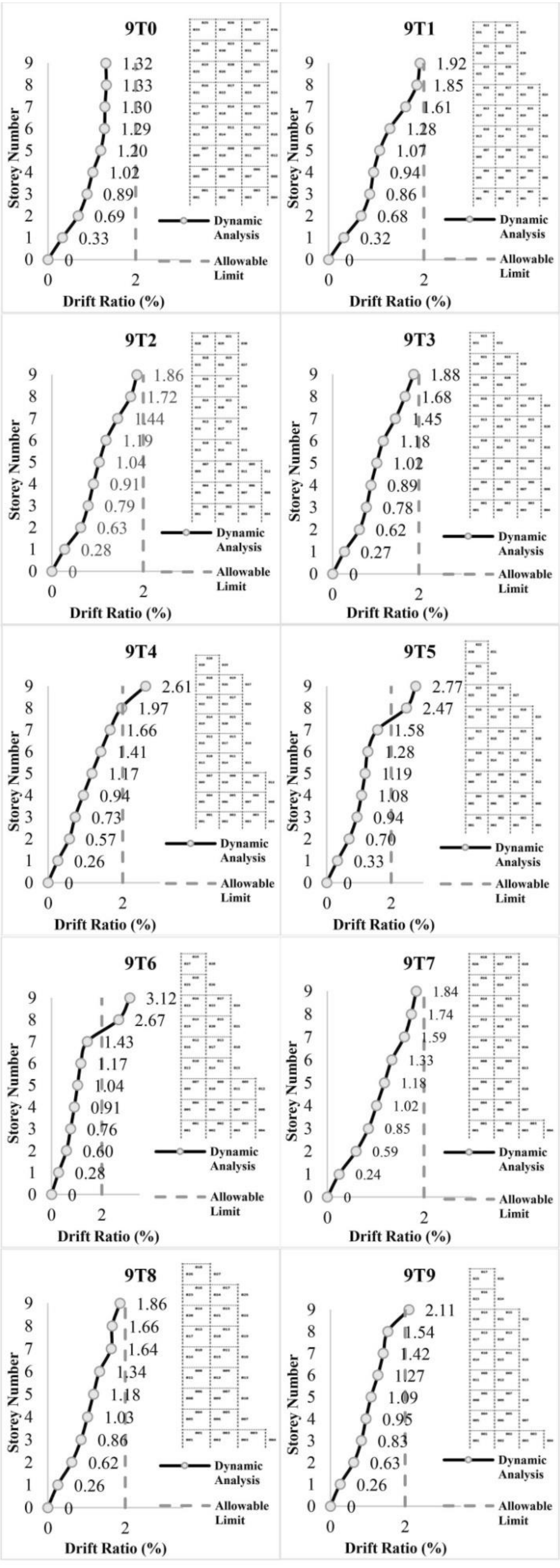


Fig. 7 Interstorey drifts ratios for 9 storey frames

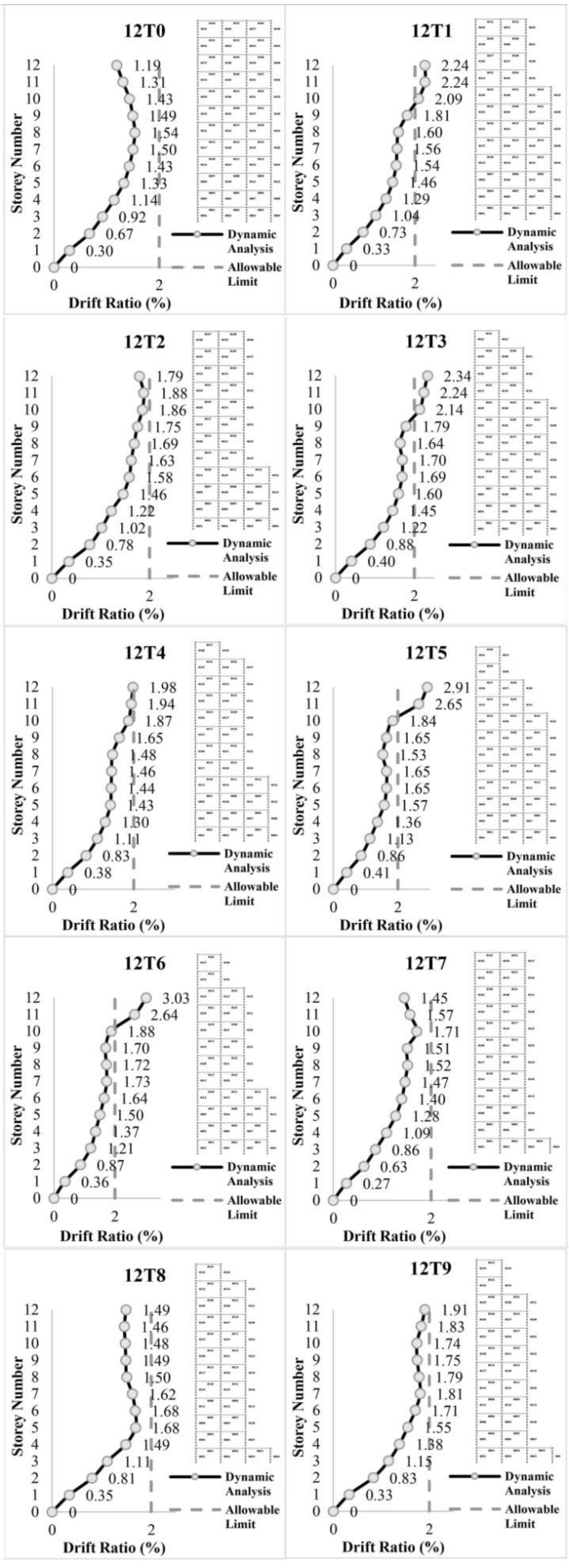


Fig. 8 Interstorey drifts ratios for 12 storey frames

hysteretic model. The hysteretic model incorporates stiffness degradation, strength deterioration, non-symmetric response, slip-lock, and a tri-linear monotonic envelope. The model traces the hysteretic behaviour of an element as it changes from one linear stage to another, depending on

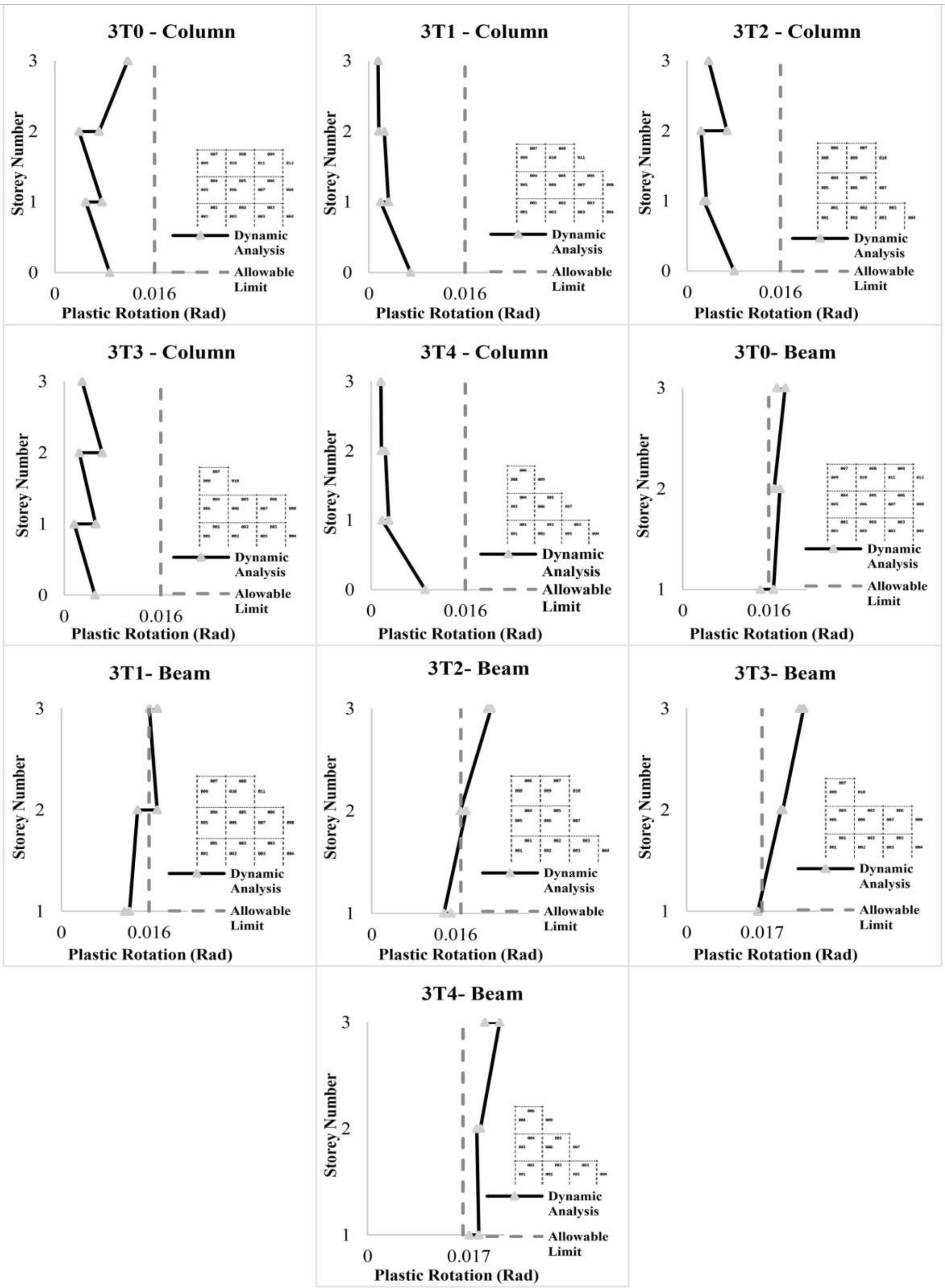


Fig. 9 Plastic hinge rotation ratios in 3 storey frames

the history of deformations. The model is therefore piece-wise linear. Each linear stage is referred to as a branch. Fig. 4 shows the influence of various degrading parameters on the shape of the hysteretic loops. For a complete description of the hysteretic model see Park *et al.* (1987). The nonlinear dynamic analysis is carried out using a combination of the Newmark-Beta integration method, and the pseudo-force method. The solution is carried out in incremental form. The dynamic input is given as a ground acceleration time-history which is applied uniformly at all the points of the base of the structure. P-Δ effects are considered in the nonlinear analysis.

4. Assessment of seismic performance

In order to assess the adequacy of Iranian seismic code criteria for satisfying the requirements of the Life Safety (LS) performance level, maximum inter-storey drift ratio of the structure and maximum plastic rotation of the members as global and local criteria; respectively, were evaluated according to the provisions of FEMA 356.

4.1 Performance of the structures

The inter-storey drifts ratios for the all frames resulting

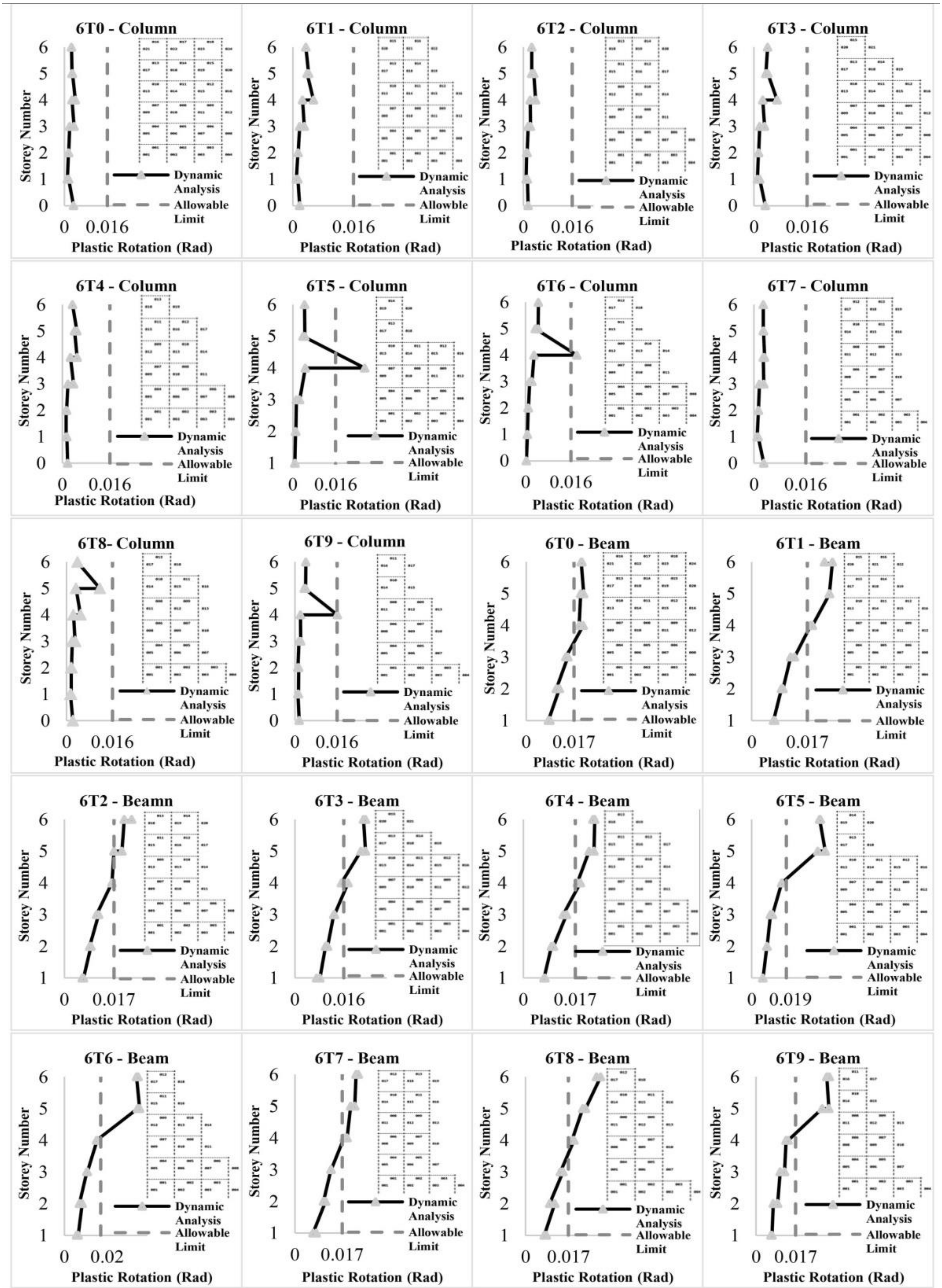


Fig. 10 Plastic hinge rotation ratios in 6 storey frames

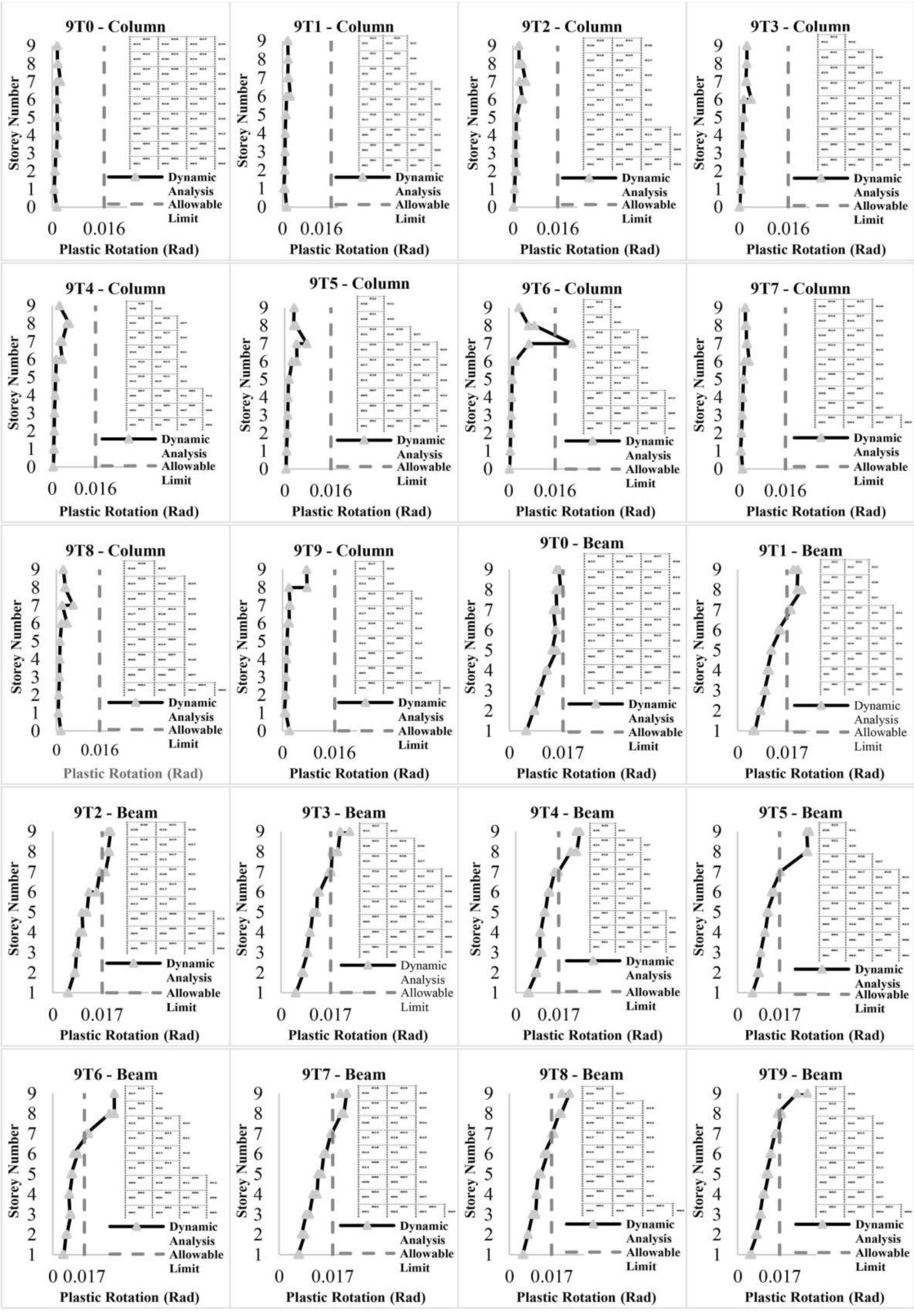


Fig. 11 Plastic hinge rotation ratios in 9 storey frames

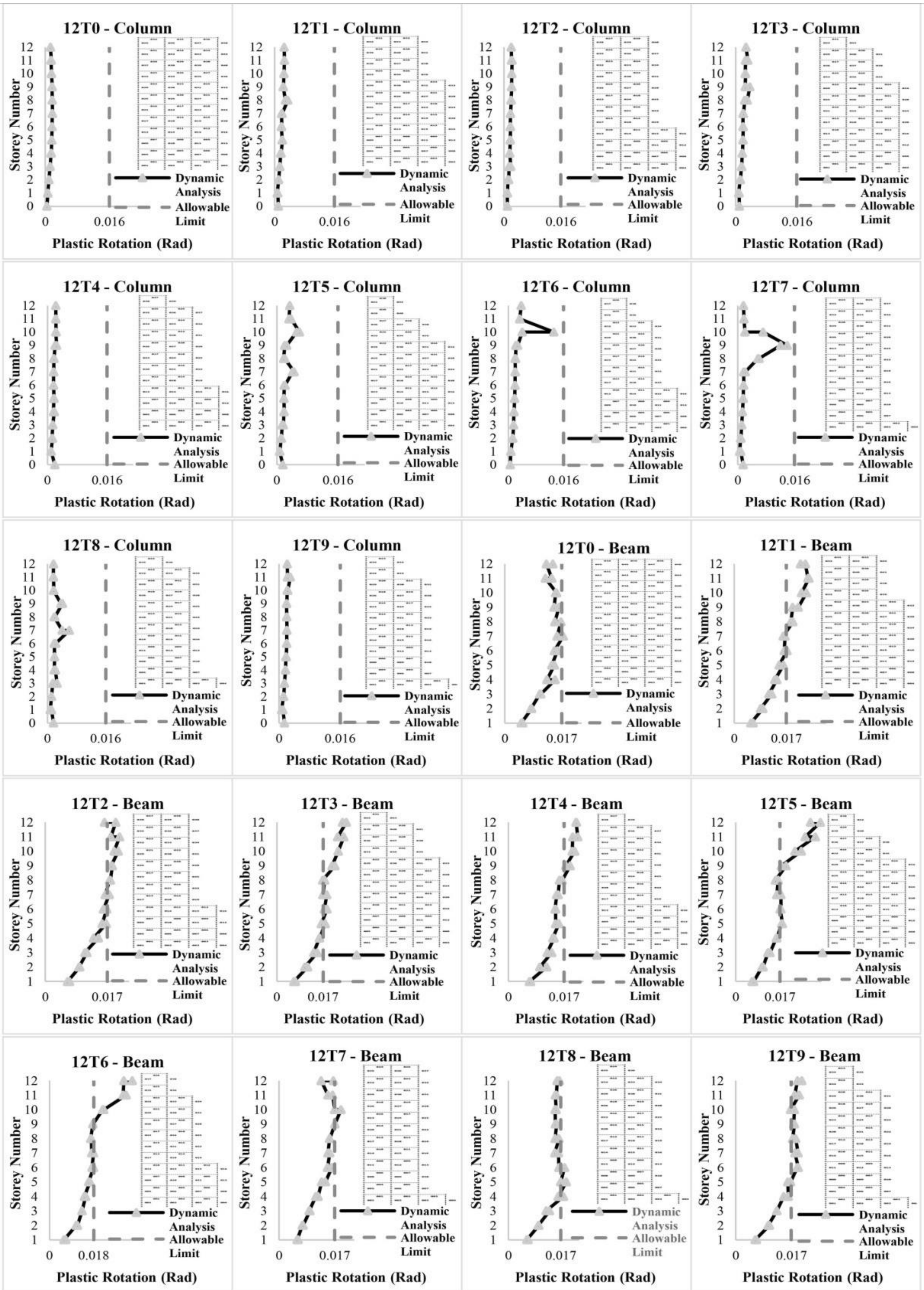


Fig. 12 Plastic hinge rotation ratios in 12 storey frames

from nonlinear dynamic time-history analyses have been summarized in Figs. 5-8. As can be seen, the regular frames (3T0, 6T0, 9T0, 12T0), satisfy the requirements of the LS performance level (limiting drift 2%) while inter-storey drifts of the irregular frames are quite different. In some of them such as 3T1, 3T2, 3T3, 6T2, 6T2, 9T1, 9T2 & etc, the inter storey drift ratios are less than limiting drift 2% and some of them could not pass the life safety performance level (3T3, 6T1, 6T3, 6T4 & etc). Habibi and Asadi (2014) revealed that most setback frames designed based on previous edition of Iranian seismic code (3rd edition 2005), could not pass the requirements of the Collapse Prevention (CP) performance level (limiting drift 4%); but this study indicated although most of setback frames cannot satisfy LS performance level, the last edition of Iranian seismic code (4th edition 2013) has been improved to prevent occurring inter storey drift ratio more than CP performance level limit. Furthermore, fewer of setback frames designed based on the 4th edition could not meet life safety performance level comparing with previous edition of Iranian seismic code.

4.2 Performance of the structural members

Figs. 9-12 show the plastic hinge rotation ratio of the ends of members resulting from nonlinear analysis comparing with their corresponding allowable values under the design earthquake. The allowable limit of plastic rotation for the LS performance level was determined for each member dependent on its action, geometric characteristics, reinforcement and type of loads according to the criterions of FEMA 356. In the figures related to columns, the rotation values at each storey level represent the maximum plastic rotation of column ends at same storey level. Also, in the figures related to beams, the rotation values at each storey level represent the maximum plastic rotation of beam ends at same storey level. As can be seen in the figures, almost all columns of frames (except 6*T*5, 6*T*6 & 9*T*6 frames) experience the plastic rotation less than LS limit while in the all of frames except 9T0 & 12T0, especially in the upper stories the plastic rotation of beams has exceeded than the LS performance level. It is clear that when setbacks occur, the local performances are affected and obviously by increasing severity of setback along height of the frames (such as 6*T*5, 6*T*6, 6*T*9, etc.), the requirements of the LS level are not satisfied. The notable point is that the maximum damages have happened at the members located in the vicinity of the setbacks. It is obvious that these members must be strengthened to satisfy the local performance criteria. By comparing this results with the last study about seismic performance of setback frames designed based on 3th edition of Iranian seismic code (Habibi and Asadi 2014), it is revealed that the local performance level of frames has been modified by current edition of Iranian seismic code so that the LS performance level in fewer members has been violated and although none of the members have not experienced the damage more than collapse perversion level but still more modification to strength the members of setback frames is necessary.

5. Conclusions

In order to investigate the seismic performance of RC setback frames, several regular and irregular multistorey reinforced concrete moment resisting frames designed based on Iranian seismic code, were analysed using nonlinear dynamic time history analysis. Results clearly state that the last edition of the Iranian seismic code has improved the seismic performance of setback frames, but the recent modified regulations cannot provide the life safety performance level for these frames yet. Therefore, in order to improve the seismic behaviour of reinforced concrete moment resisting setback buildings, the current edition needs to be modified. In addition, it was observed that the life safety performance criteria are satisfied in the regular frames such as previous edition

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